The Design and Construction of a Bridge with MSE Abutments on Seismically Liquefiable Ground

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Abstract: The Tahuna Road Bridge is part of the State Highway 1 Rangiriri to South of Ohinewai Four-Laning project south of Auckland. The bridge crosses the future motorway over seismically liquefiable ground near Ohinewai. Mechanically stabilised earth [MSE] abutments provide shallow founding for the bridge. A low-displacement Y-shape vibratory probe was used to densify the sandy soil underlying the bridge site to a depth of 6~8m to satisfy the seismic design requirements of the Transit New Zealand Bridge Manual. It is the first time that resonant vibratory compaction technique using a low-displacement probe has been employed on a relatively large scale for a bridge project in New Zealand. The successful application of a resonant vibratory probe on the project demonstrated that it is significantly more economical than other ground improvement methods commonly used in the country. The paper presents the design considerations and construction methodologies for the ground improvement and MSE abutments works. Construction aspects are also discussed.

INTRODUCTION

The Tahuna Road Overbridge forms part of the Transit New Zealand State Highway SH1 Rangairiri to South of Ohinewai Four Laning (RSO4L) design-construct contract which was awarded to the Fletcher Higgins Joint Venture.

The site is situated near Ohinewai about 70km south of Auckland and 30km north of Hamilton. The overbridge is a slightly skewed two 19m spans bridge designed to carry 2 lanes of rural traffic over the new four lane SH1 currently under construction. The site is underlain by an alluvial deposit of over 40m thick which is susceptible to seismic liquefaction.

Fletcher Higgins, on the advice of Peters and Cheung Ltd acting as a sub-consultant to the lead consultant, Sinclair Knight Merz, made the decision to use the resonant vibratory compaction technique and a Y-shape low-displacement vibratory probe for the first time on a large scale in New Zealand to treat the liquefiable bridge foundation soils. The foundation improvement works, combined with the use of spread footing and two 8m high mechanically stabilised earth (MSE) bridge abutments, provide founding for the bridge and eliminate the need to pile foundations. A typical cross section of the bridge is shown in Figure 1.

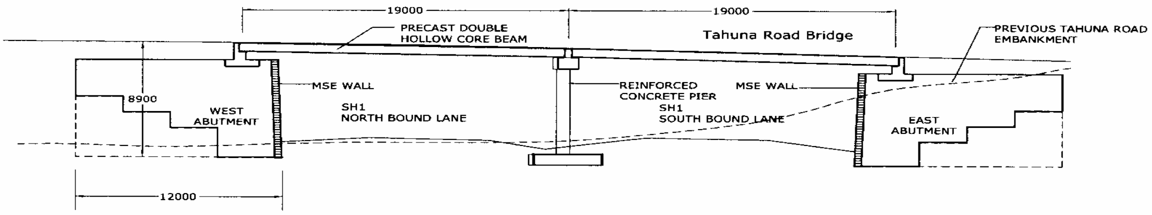

Figure 1 Typical Cross Section of Tahuna Road Overbridge Crossing State Highway SH1

The resonant vibratory compaction technique was found to be effective and economical for the ground conditions at the bridge site. The ground treatment of 1700m^2 area to 6~8m depth was completed within 18 days. The two 8m high Keystone MSE bridge abutments reinforced by Keystrip steel ladders were founded within the densified area. Extra seismic safety measures in addition to the New Zealand Bridge Manual requirements were incorporated into the bridge and abutment design by adopting the Limited Permanent Displacement Design Concept. The paper presents the design considerations and construction methodologies for the ground improvement and MSE abutments works.

GROUND CONDITIONS

The site is underlain by over 40m thick of alluvial deposits with firm to stiff silt and clay generally present below 18m depth. The upper 6m of the soils are relatively recent, loose, fine gained, pumiceous sands. A layer of medium dense pumiceous sands which slightly increases in strength with depth is present between the upper loose sand and the lower silt and clay layers. Table 1 presents a summary of the typical soil profile of the site and the soil properties in terms of cone penetration resistance, qc, and friction ratio, R_F. Ground water surface is present at about 1.5m below the ground surface in summer lifting to within 0.5m in winter.

Depth (m)	Soil Description	CPT Measurement
0 - 3m	Light brown, fine grained, pumiceous sand	qc = 1 ~ 3MPa R_F = 0.5<~ 0.8%
3 - 6m	Grey, fine grained, loose, pumiceous sand interbedded by thin layers of sandy silt between 4.5m and 6m	qc = 2 ~ 3MPa R_F = 1<~ 1.5% (Generally 1%, except thin silt layers > 2%)
6 - 8m	Grey, dense, pumiceous sand	qc = 8 ~ 10MPa R_F = 1%
8 - 18m	Grey, medium dense, pumiceous sand	qc = 4 ~ 10MPa R_F = 0.5%
Below 18m	Firm to stiff silt and clay	qc = 1 ~ 3MPa R_F = 3 ~ 4%

Table 1 - Typical Soil Profile at Tahuna Bridge Site

SITE SEISMICITY

At the request of Peters and Cheung Ltd, the Institute of Geological and Nuclear Sciences undertook a site specific seismicity assessment of the bridge site. Ohinewai is located about 30km north of Hamilton and is located within a moderately low seismic region in New Zealand. Key findings from the assessment were that (i) the site is not susceptible to fault movement related earthquake shaking and (ii) the seismicity of the site for the 475 year and 700 year return period earthquakes is similar and is mainly contributed from earthquakes with local magnitudes in the range of M_L 5.25 to 6. Furthermore, the estimated peak ground accelerations, a_{max}, are in the range of 0.1 ~ 0.2g.

SELECTION OF SEISMIC DESIGN CRITERIA

In accordance with Transit New Zealand Bridge Manual, the Tahuna Overbridge is considered as a motorway structure. In the project, we have chosen the following seismic design criteria for the design of the bridge structure and its abutments.

(i) Serviceability Limit State - M_L 6 & a_{max}= 0.12g, 150 years earthquake return period
(ii) Ultimate Limit State - M_L 5.5 & a_{max} = 0.22g, 475 years earthquake return period
(iii) 1000 years earthquake return period - M_L 6 & a_{max} = 0.22g

LIQUEFACTION ASSESSMENT

Summarised in next page are the key findings from our liquefaction assessment of the project site.

- Serviceability limit state
  - The site is not susceptible to seismic liquefaction during earthquakes with a 150 year return period.
- Ultimate limit state
  - Under no surface surcharge loading condition, the pumiceous sands at 3m-6m and 8m-14m depths are subject to liquefaction or exhibit a marginal factor of safety against liquefaction.
  - Under a 8m high bridge abutment surcharge loading condition, the entire soil profile exhibits a factor of safety against liquefaction, FoS, of generally above 1.5.
- 1000 years earthquake return period
  - Under no surface surcharge loading condition, the soil layers at 3m-6m and 8m-14m depths are subject to liquefaction.
  - Under 8m high bridge abutment surcharge loading condition, the soil layer at 3m-6m depth is subject to liquefaction while the material at 8m-14m generally has a FoS in the range of 1.2-1.5 against liquefaction.

The above results suggested that ground improvement is required to minimise the effect of liquefaction on the performance of the bridge structure and the abutments. Methods for liquefaction assessment for deriving above conclusions are presented below.

Estimation of Liquefaction Resistance of Soils

Figure 2, NCEER (1997), presents a commonly used CPT cone penetration resistance-based chart for the estimation of liquefaction resistance of clean sands with less than 5% fines content. The data in the chart presents a co-relationship between the capacity of the soil to resist liquefaction, expressed in terms of the Liquefaction Cyclic Resistance Ratio (CRR_1) and the Normalised Cone Resistance (qc_1) for earthquakes with local magnitude (Richter magnitude), M_L, of 7.5. The subscript 1 following CRR and qc denotes their, respectively, equivalent values at a vertical effective stress σ'_v of 1 atm unit (1atm $\approx$ 100kPa). For normally consolidated sand deposits, (qc_1) can be calculated from (qc_1) = qc / $(\sigma'_v)^{0.7}$. It is noted that both qc and σ'_v are given in terms of atm units.

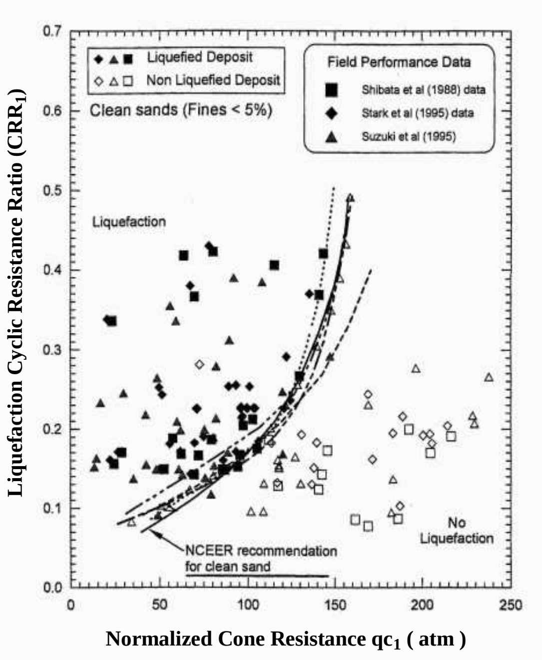

Figure 2 CRR_1 vs qc_1

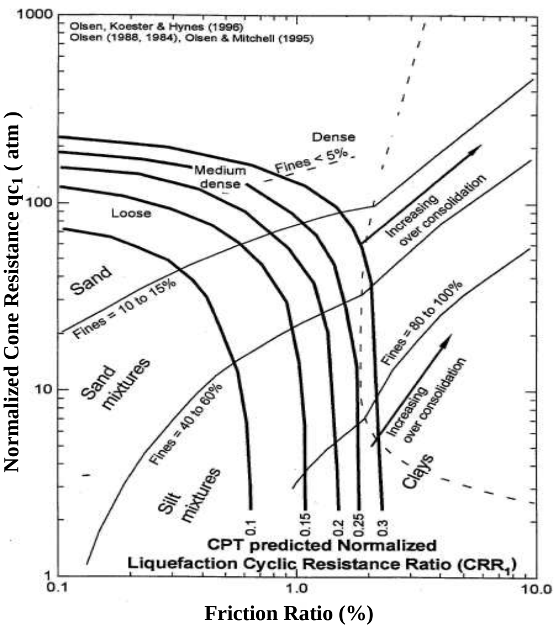

Figure 3 CRR_1 based on Both qc_1 and R_F

Artificial boundary lines proposed by various researchers to approximately separate the liquefaction and non-liquefaction soil deposits were also compared in Figure 2. All lines generally agree well with each other, except where CRR_1 is larger than 0.25. The boundary line proposed by NCEER (1997) is now commonly adopted in engineering practise.

Both cone penetration resistance qc and friction ratio R_F are measured as a routine part of CPT testings. However one of the shortcomings of Figure 2 is that the effects of the friction ratio of a soil on its liquefaction resistance cannot be assessed from the chart because it is entirely based on the measured qc values.

Figure 3 presents a CPT Soil Characterisation Chart proposed by Olsen et al (1996) for the estimation of CRR_1 in terms of both qc_1 and R_F values. It shows that the CRR_1 of a soil increases rapidly as its friction ratio increases. The chart indirectly includes the effects of soil types and the influence of fines content. Thus it is suitable for estimating the liquefaction resistance of both clean sands and soils with high fines contents. Figure 3 was also adopted for the liquefaction assessment of the bridge site.

Earthquake Magnitude Scaling Factors

Figures 2 and 3 were developed from data collected from sites where liquefaction did or did not occur during earthquakes with magnitude near M_L 7.5. For earthquakes with a magnitude smaller or larger than M_L 7.5, it is necessary to scale up or down the CRR_1 values estimated from Figures 2 and 3 by a Magnitude Scaling Factor (MSF) initially introduced by Seed and Idriss (1982). However, later research considered that the MSF values proposed by Seed and Idriss in 1982 were too conservative, and subsequently NCEER(1997) suggested that the upper and lower bound MSF values in Table 2 be adopted.

Magnitude	**5.5**	**6.0**	**6.5**	**7.0**	**7.5**	**8.0**
Upper bound	2.8	2.10	1.60	1.25	1	Uncertain
Lower bound	2.2	1.76	1.44	1.19	1	0.84

Table 2 Earthquake Magnitude Scaling Factors

Factor of Safety against Liquefaction

During an earthquake, the average shear stress, t_{av} within the soil caused by the earthquake is represented by the cyclic shear ratio, CSR, calculated from the following simplified equation.

$$CSR = t_{av}/\sigma'_v = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\ \sigma'_{vo})\ r_d \qquad (1)$$

where a_{max} and g denote the peak horizontal ground acceleration and the gravity, respectively, while σ_{vo} and σ'_{vo} represent the total and effective overburden pressures, respectively. r_d is a stress reduction factor for flexibility derived from the soil as a correction for assuming total rigidity of the overlaying soil. Alternatively, t_{av} may be calculated using a computer program such as SHAKE by modelling the soil profile with appropriate soil parameters with appropriate seismic input records.

The factor of safety, FoS, of a soil against liquefaction can be calculated from CRR, CSR and MSF as follows:

$$FoS = (CRR_{7.5} * MSF) / CSR \qquad (2)$$

where $CRR_{7.5}$ is the CRR_1 determined for magnitude 7.5 earthquake using either Figure 2 or 3.

Seismic Induced Excess Porewater Pressure

During seismic excitation, excess porewater pressure builds up within a soil regardless of whether liquefaction occurs or not. For many soils, the seismic induced excess porewater pressure, $u_{G,}$ can be expressed by Equation (3), Seed et al (1975).

$$u_G / \sigma'_{vo} = (2/\pi)\ \sin^{-1}(N / N_L)^{1/2\theta} \qquad (3)$$

where θ is in the range of 0.5 ~ 0.9 and 0.7 is usually adopted. N_L is the number of uniform stress cycles causing initial liquefaction of the soil at a given CSR, cyclic shear ratio. N is the accumulated number of significant stress cycles during an earthquake. To enable us to calculate the seismic induced excess porewater pressure from Equation 3, the N_L value of the soil need to be determined. We have compiled the CRR_1 and qc_1 relationship from Figure 2, and MSF values from Table 2 into Figure 4 so that N_L values of soils with various cone resistance subject to various levels of cyclic stress ratio can readily be calculated. Figure 4 also presented the number of equivalent stress cycles, N_{Eq}, of various earthquake magnitudes proposed by Seed et al (1975). It should be noted that if N_{Eq} of a given earthquake exceeds N_L, the soil will subject to liquefaction.

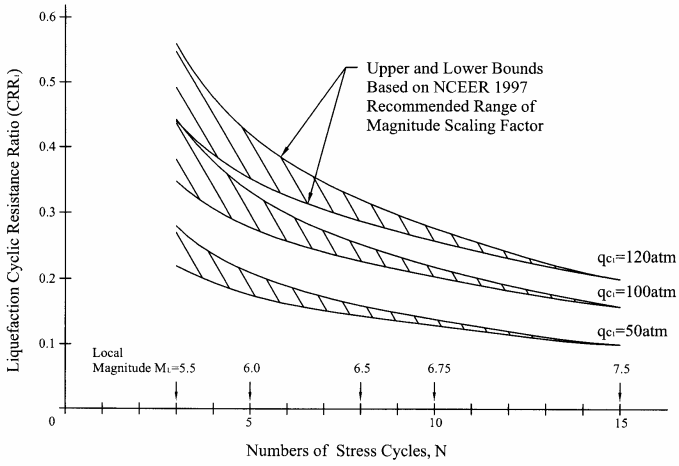

Figure 4 Relationship between Liquefaction Cyclic Resistance and Number of Stress Cycles

DESIGN PHILOSOPHY

During the tender design stage, two bridge options and four ground improvement techniques were considered. It was found that the two span option with the bridge deck founded on shallow strip foundations located immediately behind the MSE abutments and a central pier founded on a strip foundation was the cheapest option, Figure 5. The significant design features comprise the following:

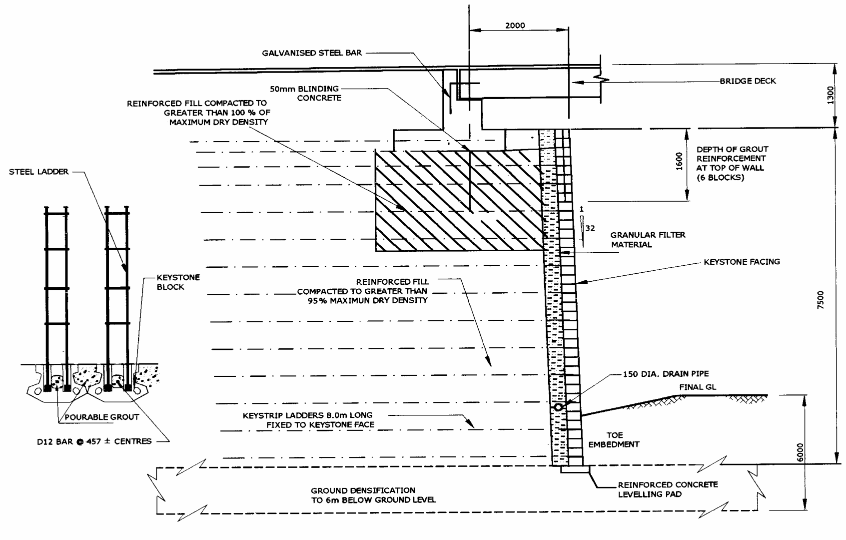

Figure 5 Tahuna Bridge Steel Ladder Reinforced Abutment with KeyStone Facing

- Ground Improvement - Extensive analyses indicated that ground improvement of the upper 6m above the non-liquefiable dense sand layer at 6-8m depth extending to an the area 5m outside the bridge and abutment footprint, Figure 6, was sufficient to satisfy the seismic design criteria outlined in Section 4 and the New Zealand Bridge Manual requirement. The improvement target was to densify the upper 6m soil to exhibit a minimum FoS of 2 against liquefaction for the 1000 year earthquakes, (M_L 6 & a_{max} = 0.22g). The minimum 6m deep improvement was also required to maintain the static stability of the abutments following severe earthquakes during the period that the seismic induced pore water pressure had not been fully dissipated. The improvement strategy was to form a 8m thick non-liquefiable soil raft underlying the entire bridge site.

- Resonant Vibratory Compaction ground improvement - The insitu sand has a fines content of 5-7%. Vibratory compaction is suitable for the soil and is the cheapest option because the Y-shape vibratory probe, comprising three 10m long steel plates, Figure 7, could be economically made for the project. The contractor had a suitable vibration hammer readily available. The resonant frequency of the ground can be measured to optimise the densification efficiency.
- Mechanically Stabilised Earth (MSE) Bridge Abutments - Inextensible MSE abutment walls have demonstrated excellent seismic performance, exhibit extremely good ductile behaviour and also have a pleasing appearance. High tensile strength geotextiles are also laid below the steel reinforcement zone to enhance the structural ductility of the abutments.
- Bridge foundations - The bridge is founded directly on strip footings at the two MSE abutments and the central pier. The bridge footings at the abutments have approach slabs and are tied into the abutment fill with steel ladders to ensure that the bridge responds elastically under the design seismic loadings. Transverse seismic loads are be resisted by friction between the abutment footings and approach slabs and the underlying granular material in the abutment fill.
- Bridge Deck - The deck comprises prestressed double hollow core beams connected with transverse prestressing designed to distribute traffic and parapet impact loadings. The two spans and the central pier and the abutment footings are tied together by galvanised reinforcement.
- Limited Permanent Displacement Design Concept - The bridge and abutment design extended the ductility concept within the framework of the Ultimate Limit State design for structures to include foundations on sands subject to seismic influence. Significant investigation of the bridge deck and seat interface was undertaken to evaluate the acceptable level of seismic induced movement of the abutments and their influence on the bridge deck. If the Ultimate Limit State design earthquake loading is significantly exceeded, (for example the 1000 year earthquake is exceeded) the abutment movement is limited to less than 200mm. The passive soil wedge behind the bridge seat is designed to fail and act as an energy absorption mechanism to dissipate the earthquake energy transferred from the bridge deck to the abutment so that the integrity of the entire bridge structure is maintained.

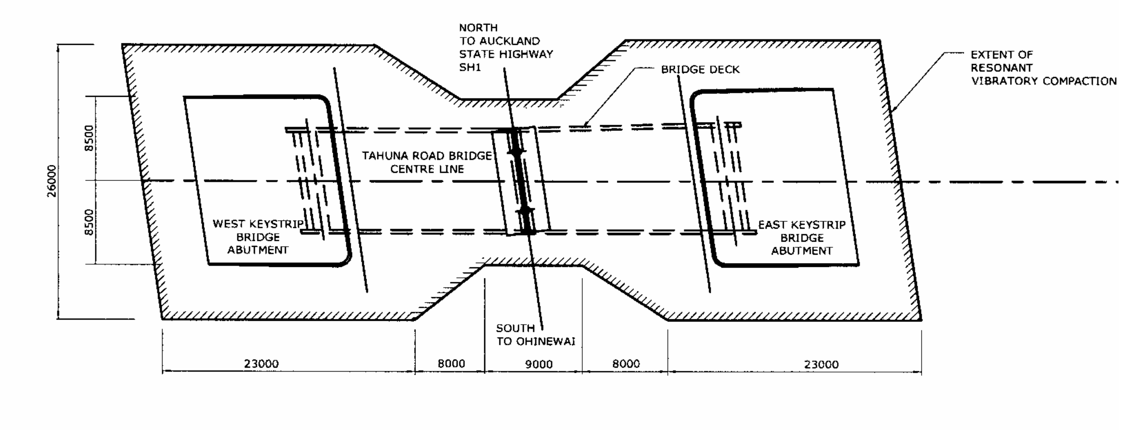

Figure 6 Resonant Vibratory Compaction Area

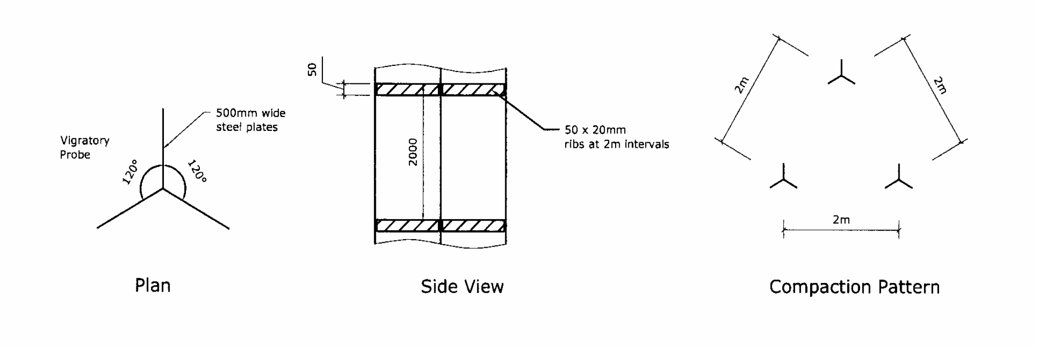

Figure 7 Vibratory Probe Configuration and Compaction Pattern

GROUND IMPROVEMENT OPTIONS

Four ground improvement techniques including (a) stone columns, (b) dynamic compaction, (c) dewatering, excavation and re-compaction, and (d) resonant vibratory compaction were considered. The densification covers an area of 1700m^2 extending 5m outside the bridge and MSE abutment footprints as shown in Figure 6. The costs of the four options range from \$78k to \$250k. The resonant vibratory compaction method was selected because (i) it was the cheapest option, (ii) it required the shortest construction time and (iii) it had very little environmental effects outside the densification area.

DESIGN METHODS

Resonant Vibratory Compaction Densification

Resonant vibratory compaction is generally suitable for granular soils with a fines content less than about 10% or CPT friction ratio lower than 1%. The soil within the densification zone is generally within this category. The technique has been employed overseas. Some background information can be found in Neely et al (1991) and Van Impe et al (1994). For soils with higher fines contents, vibro-replacement such as stone column densification may be needed.

The configuration of the Y-shape vibratory probe is shown in Figure 7. The probe comprises three steel plates of 500mm wide, 16mm thick and 10m long welded together at 120 degrees. It has 300mm long x 50mm high x 20mm thick ribs welded to both side of the plates at 2m vertical intervals.

The compaction points were arranged in an equilateral triangular pattern at 2m centre to centre spacing covering the entire densification area. A 4 tonne variable frequency vibration hammer was selected for driving and withdrawing the vibratory probe from the ground during the ground improvement process. For the selected 2m compaction point spacing, the surging method was found to develop the most effective compaction effect during the initial field trials. This involved inserting and extracting the probe 5 times to its 6-8m full depth at a rate of 2m per 15 second, with one minute steady state vibration at its full depth before each withdrawal.

MSE Abutments

The seismic induced porewater pressure distribution within the soils beneath and outside the bridge abutment areas caused by the serviceability and ultimate limit states, and the 1000 years return period earthquakes as given in Section 4 were calculated using the methods presented in Section 5. We assumed that the effective angle of friction was $\Phi' = 35^o$ for the upper densified pumiceous sands. Below the 8m depth, the effective angle of friction for the unimproved soils was assumed to be $\Phi' = 30^o$. We also assumed that the total density of the unimproved pumiceous soil was 16kN/m^3 and that it increased to 18kN/m^3 after the densification.

Using the calculated seismic induced porewater pressure distribution in addition to the static groundwater pressure and the assumed soil parameters, a series of stability analyses were undertaken for the abutment design. The abutments were designed to exhibit a minimum factor of safety of 1.5 to maintain the static stability of the abutments following the ultimate limit state design earthquakes during the period that the seismic induced pore water pressure had not been fully dissipated. Our model predicted that no/insignificant seismic induced abutment movement is caused by the design earthquakes.

For the 1000 year design earthquake, the abutments are designed to have a static factor of safety of at least 1.3 after the seismic induced pore water pressure is fully developed within the soil. We have conservatively estimated that the seismic induced permanent abutment movement is unlikely to exceed 200mm. The passive soil wedge behind the bridge seat is designed to fail and act as an energy absorption mechanism to dissipate the earthquake energy transferred from the bridge deck to the abutment so that the integrity of the entire bridge structure is maintained.

CONSTRUCTION

Resonant Vibro-compaction Field Trial

At the beginning of project, field compaction trials were undertaken partly within an area previously pre-loaded by the old 3m-6m high Tahuna Road Rail Overbridge embankment which was removed prior to the ground improvement work. At the time of the trial, groundwater level was at 1.3m below the ground surface.

To determine the natural frequency of the ground and to maximise the compaction efficiency, the vibratory probe was inserted into and extracted from the ground by the vibration hammer at various rotational frequencies. Vibration monitoring was undertaken at a 2.5m distance from the centre of the vibratory probe on the ground surface. Maximum ground response was measured at about 20Hz (0.05 second) corresponding to the rotational frequency of the vibration hammer at 1600-1700rpm (26Hz). Visually, when the vibratory probe was operated at the 1600-1700rpm hammer rotational frequency, the induced ground settlement and the influence zone of the probe is significantly larger than those induced at other rotational frequencies. It was therefore decided that all subsequent compaction would be carried out at 1600-1700rpm hammer frequency.

During the initial trials, for the selected 2m compaction spacing, it was found that the vibratory compaction surging method as discussed in Section 8 was the most effective method. During the 5 repeatedly surging and steady state vibration cycles, the ground within a 1.5m radius from the centre of the probe settled by 250mm. Subsequent compaction at the surrounding compaction points caused additional 100mm settlement making a total of 350mm settlement.

Spectral Analyses of Surface Waves (SASW) tests were undertaken immediately after the compaction trials to determine the changes in the shearwave velocity properties due to the resonant vibratory compaction. It was generally found that the vibratory compaction immediately increased the shearwave velocity from 80m/sec to 125m/sec. It is equivalent to an improvement of the small strain shear modulus, G, by a factor of 2.

Ground Improvement Results

Ground settlement induced by the vibratory compaction was surveyed as the densification work was in progress. It was observed that the compaction induced settlement characteristics of the ground were different in the areas inside and outside the previous Tahuna Road Rail Over-Bridge embankment footprint.

In the area directly under the previous 6m high embankment at the eastern end of the site, the densification process induced settlement in the order of 250mm. In areas outside the removed embankment, the induced settlement was in the order of 600mm. The difference in their characteristics is due to the pre-consolidation history of the ground.

Depth (m)	Before Densification	After Densification
0 - 3m	qc = 1 ~ 3MPa R_F = 0.5<~ 0.8%	qc = 3 ~ 5MPa (Typical 4MPa) R_F = 1 ~ 1.5% (Up to 2%)
3 - 6m	qc = 2 ~ 3MPa R_F = 1<~ 1.5% (Generally 1%, except thin silt layers >2%)	qc = 3 ~ 4.5MPa R_F = 1.3 ~ 2.0% (Little improvement of qc and R_F in silt layers)

Table 3 Comparison of CPT Test Results Before and After Densification

40 CPT tests were undertaken as part of the QA requirement for the verification of the ground improvement. We have summarised the CPT test results before and after the vibratory compaction in Table 3. Generally, after the compaction, all treated soils exhibited a minimum FoS of 2 against liquefaction for the 1000 year return period earthquake, except for a few thin layers at two CPT

locations where the FoS is slightly below 2. Our detailed analyses of the CPT results using Figure 3 indicated that the increase in the friction ratio due to the compaction could contribute to the improvement in seismic liquefaction resistance of the treated soil as much as the increase in the cone resistance does. Furthermore, the results generally suggest that very little increase in both cone resistance and friction ratio for materials with high silt and clay contents, say F_R>2%.

MSE Abutment Wall Settlement

During the construction of the MSE abutments settlement pins were installed and settlement of the abutments were monitored during and after construction of the bridge. The 8m high eastern abutment is partly located within the previous 6m high Tahuna Road Rail Overbridge embankment area. The total settlement measured was 50mm.

The western abutment is located outside the previous embankment area. A total settlement of 100mm was measured. The majority of the settlement occurred during the 4 week MSE wall construction period. This enabled the immediate start of the bridge concrete works without delays to allow settlement.

The reconstructed Tahuna Road rail embankment adjacent to the bridge site was also monitored for settlement. The embankment is about 7.5m high and was built outside the vibratory compaction zone. The embankment has settled in the order of 300mm. The result indicated that the resonant vibratory compaction has effectively eliminated about two-thirds of the bridge abutment settlement.

Central Bridge Pier Settlement

The central bridge pier is founded on a 4m wide x 10m long strip footing with a 1m embedment depth. The footing is subject to a 150kPa bearing pressure under the serviceability limit state load. The construction of the bridge deck load on the central pier induced insignificant settlement.

DISCUSSIONS & CONCLUSIONS

Key findings from the project are discussed below:

Vibratory Probe Compaction

- An extensive literature survey appears to suggest that it is the first time resonant vibratory compaction technique has been used for the densification of pumiceous sands.
- Overseas published resonant vibratory compaction data for sands mainly employs cone penetration resistance qc as a sole parameter for the evaluation of the effectiveness of the densification process. No published information on the friction ratio is available for comparison with our data. However, the friction ratio is an important parameter for the evaluation of the liquefaction resistance of soils.
- Generally, the increases in the qc values of pumiceous sands due to the densification are not as much as those in published data for non-pumiceous sands. However, with the consideration of improvement from both qc and F_R values, the resonant vibratory compaction increases the liquefaction resistance of the pumiceous sands in terms of CRR_1 at least 80~100%.
- Available data supports the view that the resonant vibratory compaction technique is only effective for materials with CPT friction ratios less than 2% or fines content less than 10%. Our measurements indicated that very little differences in both qc and F_R are noticed before and after vibratory compaction when the silt & clay contents are high.
- CPT results appear to indicate that, although the pumiceous sands are highly permeable and the ground settlement occurred immediately during the vibro-compaction stage, it took at least three days for the ground to develop its full densified strength.
- On average 30 compaction points were completed within one working day during the construction period in summer. The speed of the construction is essentially limited by the safe operating temperature of the vibration hammer which should not exceed 90^{o}C. The production rate can be significantly improved in cold or rainy days.

- The densified ground effectively reduced the 8m high bridge abutment settlement from 300mm to 100mm when compared with the induced settlement of the re-constructed Tahuna Road/Rail Overbridge embankment outside the densified zone.
- Resonant vibratory compaction was found to be an economical and environmental friendly ground improvement technique for the project site.

MSE Bridge Abutments

- The construction of the two 8m high KeyStone MSE abutments was completed in 4 weeks without the need for heavy cranage and can be considered relatively efficient when compared to other construction methods such as adding two extra bridge spans with normal bridge approach earthfill embankments.
- There are only a few contractors with sufficient experience for the construction of large KeyStone MSE walls in New Zealand and the labour cost component of MSE wall construction still appears to be relatively high.
- Compaction of the structural fill within the reinforced zone near the MSE wall face to 95% of the maximum dry density is relatively difficult and should be undertaken with extreme care. Over compaction with a heavy roller could cause distortion at the bottom of the wall face.
- The MSE bridge abutments have been designed to deal with the seismic requirements of the New Zealand Bridge Manual. Extra seismic safety using the Limited Permanent Displacement Design Concept was also introduced in the bridge design. If the Ultimate Limit State design earthquake loading is significantly exceeded, (for example the 1000 year earthquake is exceeded) the abutment movement is limited to less than 200mm. The passive soil wedge behind the bridge seat is designed to fail and act as an energy absorption mechanism to dissipate the earthquake energy transferred from the bridge deck to the abutment so that the integrity of the entire bridge structure is maintained.

ACKNOWLEDGEMENTS

Fletcher Higgins Joint Venture was the head contractor the project. Brian Perry was the specialist ground stabilisation sub-contractor undertook the ground improvement works. Sinclair Knight Merz was the lead consultant of the project engaged by Fletcher Higgins. Peters and Cheung Ltd was the specialist bridge designer responsible for the design of the bridge, abutments and ground stabilisation.

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